Chapter 6
Determination of Flood Elevations

6-1. River Hydraulics

Chapter 5 presented methods for determining a peak discharge or volume of runoff from a flood event. However, much of flood analysis and design requires the severity of a flood to be measured in terms of a depth, water surface elevation, or area flooded, rather than peak discharge. This chapter describes the general methods used to determine water surface elevation, given flow.

a. Simple versus complex. Many methods exist for making the conversion from peak discharge to flood elevation, ranging from a simple rating curve to multidimensional analysis. Each requires increased increments of time, money, and engineering experience to be successfully applied. Paragraphs 6-2 through 6-5 describe the most common methods and give a basis for proper method selection.

b. Steady versus unsteady analysis. Flood elevation analyses may be subdivided into those based on steady flow (discharge is constant with time) and those based on unsteady flow (discharge varies with time). The latter is closer to the real-world situation; however, the great majority of analyses of river hydraulics can be made assuming steady flow. Unsteady flow evaluations are considerably more complex. Paragraphs 6-3 and 6-4 describe these two types of analyses.

c. Rigid versus mobile boundary. Alluvial streams experience modifications to their geometry with time, due to sediment transport. Erosion and deposition cause increases or decreases in a stream's flow capacity, which can be reflected by changed flood elevations. However, most flood elevation determinations may be satisfactorily made by assuming that the stream boundary is rigid, greatly simplifying the river hydraulics analysis. Paragraph 6-6 discusses mobile boundary hydraulics and its application.

6-2. Development and Use of Rating Relationships

a. Gage sites. The conversion of discharge to river stage, or water surface elevation, is most accurate (and easiest) when performed at a gage. Continuous measurements of stage, along with periodic measurements of flow, serve to give a direct relationship for discharge, when the stage is known. Figure 6-1 gives an example of a stage-to-discharge relationship at a river gage. This relationship is developed by many years of data accumulation at the gage site. As seen, many points are available for discharges within banks or that slightly exceed bank-full stages. Higher discharges occur infrequently, only during floods, and only a few points in this portion of the rating curve may be available. The fewer actual data points, the more uncertain the relationship.

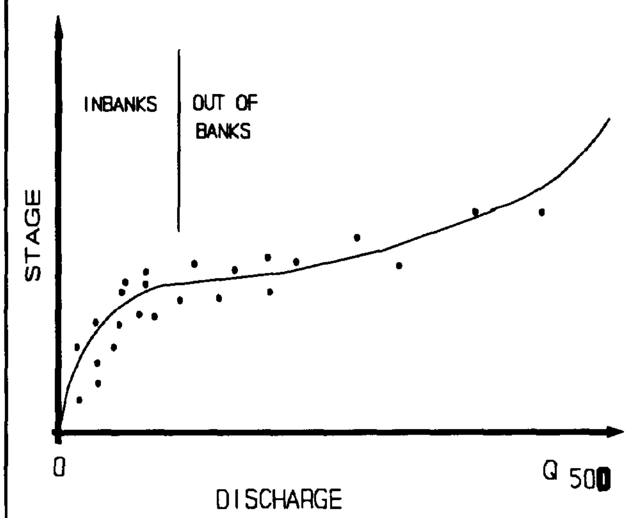


Figure 6-1. Rating curve developed from gaged data

b. Rating curves. Changes in land use, channel configuration, and boundary conditions serve to cause differences in water surface elevations for the same discharge. As mentioned earlier in this report, discharge measurements are not absolute and an error of 5 percent or so compared to the "true" discharge is not unusual. Consequently, a rating curve is usually a best-fit relationship drawn through the accumulated data points. Similar recurrences of past discharges may result in stages somewhat higher or lower than the past stages recorded.

c. Usefulness of rating relationship. As one moves upstream or downstream from a gage site, the rating relationship provides less useful information. Synthesizing a rating relationship at ungaged locations normally requires computations of water surface profiles using a computer program. Consequently, a measured rating relationship is most useful at the gaged site for calibrating a river hydraulics model to reproduce known stages for measured discharges.

6-3. Steady-Flow River Hydraulics

The use of available gaged data alone is seldom sufficient for a flood study. A flood study is normally performed for a length of stream, with flood information necessary throughout the reach, not just at a gage site. This requires the calculation of water surface elevations at many locations along the reach. This establishes a water surface elevation profile for a given flood discharge, and is usually accomplished by using a computer program. These programs assume steady, gradually varied flow with a rigid boundary. A steady flow assumption postulates that the discharge changes so slowly with time that it can be assumed to be constant for the computation period. A gradually varied flow assumption states that depth and velocity for a specific discharge change in very small increments with distance as calculations proceed along a reach of river. For the vast majority of all water surface profile computations, these two key assumptions are quite acceptable and form the basis for steady-flow river hydraulics analysis (USACE 1990b).

a. Basic principles.

(1) Given the above two assumptions, steady-flow river hydraulics analysis utilizes the conservation of mass (continuity) and energy principles (Chow 1959). Figure 6-2 shows the basic equations for computing steady, gradually varied profiles.

(2) The conservation of energy equation states that energy cannot be created or destroyed. Changes in energy levels from one point to another in the stream system occur when flowing water loses elevation in overcoming friction effects between the two points. These energy losses are primarily from boundary friction, with some additional losses due to cross-section geometry fluctuations. Changes in area and velocity at each point are calculated by the continuity equation. Velocity at each point is found by use of Manning's equation.

(3) Methods and procedures for steady, gradually varied river hydraulics analysis are well-founded and understood. However, application of the technique requires the acquisition of considerable input data.

b. Geometric data.

(1) Introduction.

(a) The geometry of the stream reach under investigation must be defined. This requires surveying and mapping work. Aerial contour mapping gives the most information on the overbank areas, with supplemental channel cross sections taken in the field. Crossing obstructions must also be described. Although acquisition of this survey data is expensive, the data have a variety of uses besides hydraulic modeling, including elevation of structures for economic analysis and topographic information for structural flood control measures.

(b) Cross-sectional locations coincide with the calculation steps of the finite difference profile analysis process. They are commonly located for the physical and hydraulic reasons listed below.

- Where distinct changes in stream bed slope occur.
- Immediately upstream and downstream of locations where changes in discharge occur.
- Where variations in geometry, including abrupt expansions and contractions in flow geometry, occur.
- Where variations in channel and overbank resistance occur.
- At bends in the stream to ensure that channel and overbank reach lengths are correctly defined.

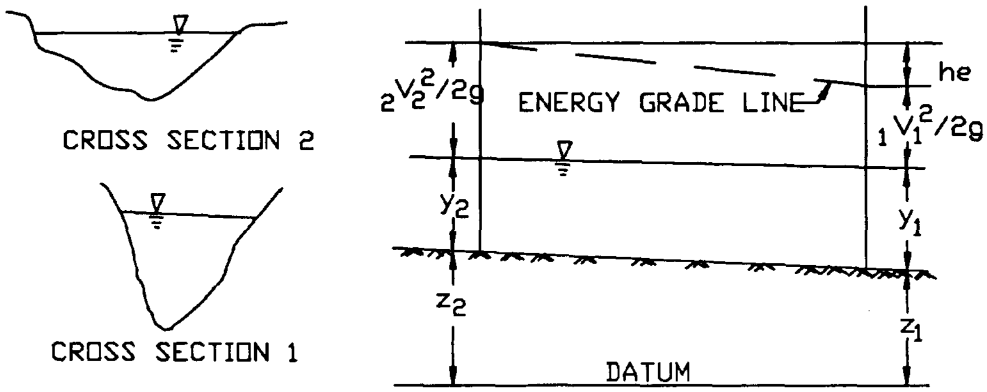
(c) Interpolated cross sections may be required to provide sufficient computation points to accurately compute the energy loss (USACE 1986).

(2) Friction loss coefficient data. Loss coefficients are determined by the hydraulic engineer from field inspection of the study reach, comparison with published references, and by engineering judgement. Friction loss coefficients (Manning's n) are often used as the main adjustment parameter to improve the calibration of the hydraulic model.

(3) Discharge data. Discharge is read from discharge-frequency relationships that are determined by hydrologic modeling or statistical analyses, as described in previous chapters.

(4) Other data. Other needed information (expansion-contraction losses, flow regime, boundary conditions, etc.) usually require minimal time and effort to develop.

(5) Calibration data. Models using gradually varied, steady-flow assumptions are calibrated to reproduce known water surface elevations with known discharges at gage sites. The main calibration technique is the adjustment of "n" values, the hydraulic parameter which contains



Continuity Equation

$$Q = A_1 \times V_1 = A_2 \times V_2 \quad \text{Equation 1}$$

Manning's Equation

$$V = (1.486/n) \times R^{0.67} \times s_f^{0.5} \quad \text{Equation 2}$$

Energy Equation

$$z_2 + y_2 + \alpha_2(V_2^2/2g) = z_1 + y_1 + \alpha_1(V_1^2/2g) + h_e \quad \text{Equation 3}$$

where

Q = discharge, cubic feet per second
A = cross-sectional area, square feet
V = average velocity, feet per second
n = Manning's coefficient of friction, dimensionless
R = hydraulic radius (area/wetted perimeter), feet
s_f = friction slope, feet/foot
z = elevation of channel invert
y = channel depth, feet
g = gravitational constant, feet/sec/sec
z + y = water surface elevation, feet above a datum (usually mean sea level)
h_e = energy loss between sections, feet

α = velocity distribution coefficient, dimensionless

$\alpha(V^2/2g)$ = velocity head, feet

Figure 6-2. Gradually varied, steady-flow equations

contains the most uncertainty. Without sufficient gaged data, the hydrologic engineer attempts calibration by reproducing high-water marks from one or more actual floods, as obtained from interviews with local residents.

c. Applications. Gradually varied steady-flow techniques are the primary method of determining flood elevations for most hydrologic analyses and have a wide applicability for Corps hydrologic studies. Common applications include:

(1) Development of flood profiles for land use planning for flood insurance/floodplain studies.

(2) Development of flood profiles for urban and rural flood damage evaluations.

(3) Determination of changes in flood elevations due to structural flood control improvements.

d. Limitations. Gradually varied, steady-flow analysis can be considered applicable as long as (1) the discharge is steady with time and gradually varied with distance, (2) the discharge can be considered one-dimensional (a single elevation for one cross section), (3) the river slope is small (less than one in ten, so a hydrostatic pressure assumption is correct), and (4) each cross section is rigid (no significant scour or deposition). When any of these assumptions is not acceptable, other techniques must be used.

e. Need for advanced analysis. The complexity of determining flood elevations increases significantly when more advanced analysis techniques are required. These techniques are usually necessary when the discharge changes rapidly with time, thereby causing flow momentum to become significant. The kinds of situations requiring more detailed computational analysis include:

(1) Dam break analysis.

(2) Flood elevation predictions at multiple points and times for very mild slopes.

(3) Where downstream boundary effects are changing, such as those caused by tidal fluctuations.

(4) Where flow is rapidly varying, such as during hydropower operations, during locking operations, sudden opening or closing of gates, abrupt start and stopping of pumping plants, and flash floods on small streams.

6-4. Unsteady-Flow River Hydraulics

The next higher level in river hydraulics computational difficulty is the application of one-dimensional unsteady, varied flow analysis. One-dimensional means that one elevation is still characteristic of each computational point, or cross section; however, now the computations are being performed at all time periods as well as all points along the center line of the river. Changes along the channel length can also be gradually varied with this technique. Figure 6-3 illustrates the difference between the results of steady versus unsteady analysis. Differences between steady and unsteady flow analysis can also be visualized by imagining one is standing on a riverbank and observing the moving water. Steady-flow analysis is adequate when the water surface appears to rise and fall uniformly, without any observation of curving streamlines. Unsteady flow analysis is necessary if one would observe an advancing wave front moving downstream, with obvious curvature to the streamlines. Figure 6-4 further illustrates this concept.

a. Hydrologic versus hydraulic routing. Unsteady flow analysis is often referred to as hydraulic routing, because elevations, velocity, and discharge information are being calculated at all time periods and for each desired location. Unsteady flow analysis can be broken into two groupings: hydrologic or hydraulic routing. Hydrologic routing is discussed in paragraph 5-5*a*. Hydraulic routing includes both continuity and momentum conservation and yields information on velocity, discharge, water surface elevation, travel times, etc. at each computational point. This section will be concerned only with hydraulic routing, or gradually varied unsteady flow.

b. Basic principles. Unsteady flow analysis is required when the inertial effects of flow, resulting in unbalanced momentum, are large enough that they can no longer be ignored. The listing of unsteady flow situations in paragraph 6-3*e* represents many of these cases. The basic equations for one-dimensional unsteady flow analysis are given in Figure 6-5. As seen, the difference between steady and unsteady flow analysis is the inclusion of the local acceleration term in Equation 2, along with the more rigorous presentation of the continuity equation in Equation 1. Solution of the unsteady flow equation is difficult and requires significant computational operations, necessitating a high-speed computer. A number of unsteady flow analysis programs are available, e.g., Fread (1978).

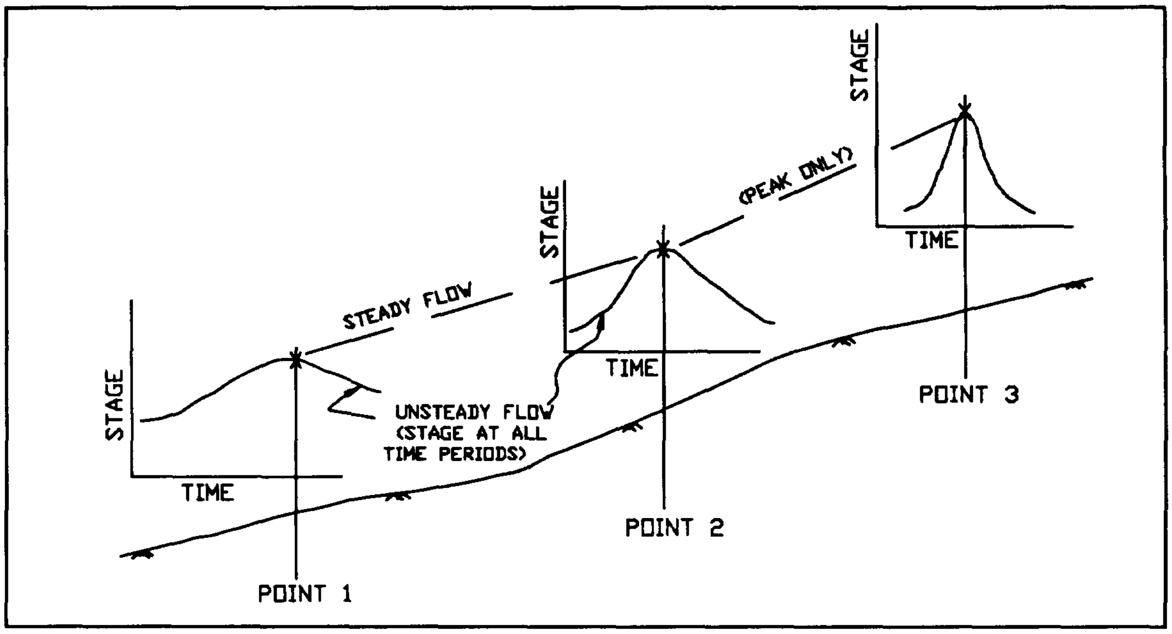


Figure 6-3. Steady- versus unsteady-flow analysis

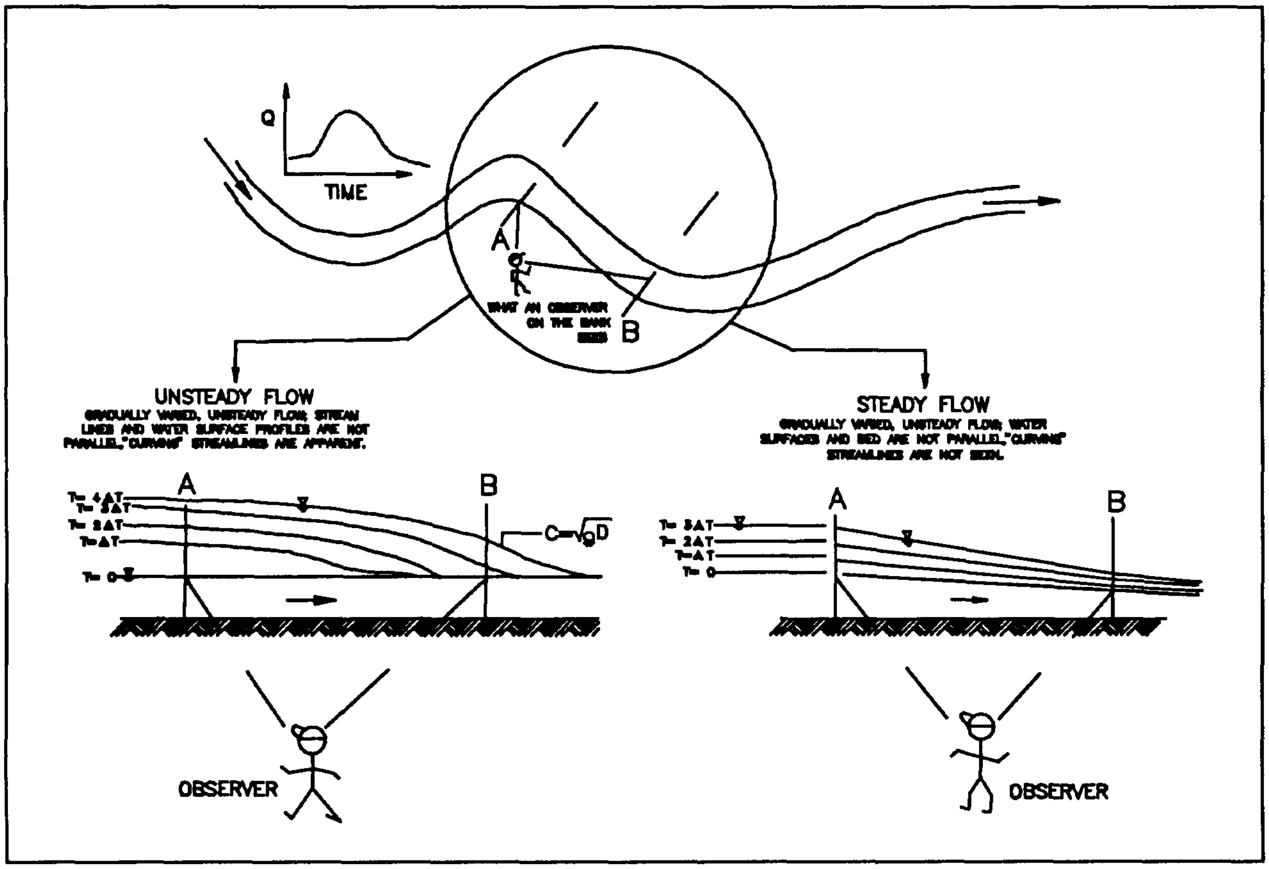
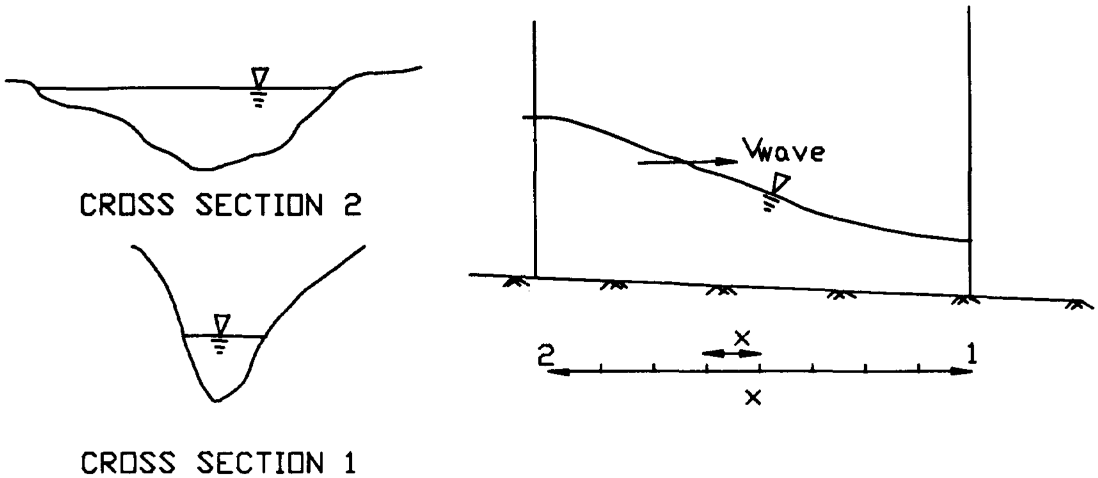


Figure 6-4. Visualization of unsteady and steady flow

Equations for a Wide Channel with No Lateral Inflow



Continuity Equation

$$y\,(\partial v/\partial x) + v(\partial v/\partial x) + (\partial y/\partial t) = 0 \quad \text{............................ Equation 4}$$

Momentum Equation

$$s_f = s_o - (\partial y/\partial x) - \frac{v}{g}(\partial v/\partial x) - \frac{1}{g}(\partial v/\partial t) \quad \text{........................ Equation 5}$$

where

y = depth, feet
v = velocity, feet per second
x = distance, feet
t = time, minutes
g = gravitational constant, feet/second/second
s_o = channel invert slope, feet/foot
s_f = friction slope, feet/foot

∂ = partial derivative, "change with respect to"; i.e., $\partial v/\partial x$ is the change in velocity with respect to distance

Figure 6-5. Unsteady, gradually varied flow equations *c. Data requirements.* Requirements are similar to those of steady, gradually varied methods, with the exception of boundary conditions and calibration data. For hydraulic routing, the boundary conditions must be completely described as a stage or flow hydrograph. It is generally preferable for the stream geometry to be better defined (more cross sections) than for steady-flow methods. Calibration data for unsteady flow are also more extensive, requiring stages and/or discharges at a number of different time periods. This information is usually more costly to obtain than calibration data for steady-flow applications.

d. Applications. The common applications of one-dimensional unsteady flow analysis have been previously stated in paragraph 6-3*e*. A number of unsteady flow models have been developed in recent years and utilization of these types of models has been made easier. A higher level of engineering expertise is still necessary, however, to use these techniques. As most situations for calculating flood elevations use steady, gradually varied flow, fewer individuals are sufficiently knowledgeable and experienced to properly apply and interpret the results of unsteady-flow models.

e. Limitations. Since this method accounts for more of the physical processes that are occurring than steady-flow analysis does, there are fewer limitations. As the next level of analysis is still more complex, situations where one-dimensional unsteady flow solutions are computationally inadequate are fortunately few. Situations requiring a higher level of computational analysis include:

(1) Analysis of flow patterns in bays and estuaries, where velocities and elevations may vary in the horizontal and vertical directions.

(2) Cases in which a one-dimensional assumption cannot model the elevations with sufficient accuracy; i.e., multiple bridge openings across a wide floodplain, major river junctions, etc.

(3) Analysis of flow patterns around dike fields, hydropower plants, and cofferdams.

6-5. Multi-Dimensional River Hydraulics

Although nearly all flood elevation determination requirements can be satisfied with either one-dimensional steady or unsteady flow models, certain specialized problems occasionally require a yet more sophisticated and complex modeling approach. Use of multi-dimensional river hydraulics is necessary when one can no longer assume that a single elevation at each computational point (cross section) is appropriate. This problem requires the use of a two-dimensional (2D) model, where hydraulic properties vary across the section as well as along the length of stream, or of a three-dimensional (3D) model, which would include changes of hydraulic properties in the vertical direction. Three-dimensional computer models are currently under development and testing and are not yet fully available. Three-dimensional efforts have largely been through the application of physical models, the subject of paragraph 6-7. Only 2D modeling is addressed further in this section (EM 1110-2-1415).

a. Principles. Multi-dimensional models are usually applied to evaluate a short reach of river, where average depth is small compared to the average stream width. Because of the relative shallowness compared to length and width dimensions, differences in the vertical for hydraulic properties are often averaged to obtain a 2D solution. This greatly simplifies the work effort. The basic equations to solve 2D unsteady-flow problems are lengthy and are not included here. Assumptions inherent in the application of this technique include: gradually varied flow, constant water density, and a rigid boundary (or one that is changing insignificantly).

b. Data.

(1) General. Data requirements are considerably greater than for previous methods. It is normally insufficient to utilize a data set developed for steady or one-dimensional unsteady flow in a multi-dimensional model.

(2) Geometry. Geometry is usually derived from map data. Close interval contour mapping is most desirable, with 0.5-foot intervals often used. Since most applications of 2D models are for detailed analysis of a short reach of stream, this type of topographic information is usually feasible.

(3) Turbulent exchange coefficients. Turbulent exchange coefficients, used for modeling eddy losses, are required in addition to other coefficients such as Manning's n.

(4) Velocity. Velocity and velocity direction measurements are needed. As vertical velocities in a 2D model are depth averaged, these prototype measurements also must be depth averaged. Depth, water surface elevation, and velocity data at many points in the distance-time grid must be obtained.

(5) Acquisition. Data required for 2D models are site-specific and usually developed through a data collection program. Data acquisition is a considerable cost for 2D modeling.

c. Applications. Often, use of a multi-dimensional model requires contracting with a Corps Lab or a private consultant to develop the input data and operate the model. Considerable start-up expense and time are required to educate a new user of a multi-dimensional model, although if additional applications in the near future are foreseen, in-house capability should be further investigated. Figure 6-6 shows a typical application of 2D modeling. Other examples were indicated in paragraph 6-4*e* with additional applications, including the following:

(1) Channel deepening. Investigating the effects of deepening a ship channel on velocity patterns and shoaling.

(2) Encroachment. Investigating the effects of major encroachment into a river channel on flow patterns and water surface elevations.

(3) Velocity and flow patterns. Investigating the velocity and flow patterns of water entering and leaving a wide floodplain from the river channel.

d. Limitations.

(1) Practical limitations. The practical limitations of 2D models are in their application and in the user skills required. Because of input data needs and computational requirements, applications are normally for a short (1 mile or less) reach of river. Qualified personnel skilled in utilizing 2D models are often more difficult to obtain.

(2) Technical limitations. Technical limitations include the necessary assumptions of gradually varied flow and of insignificant changes caused by sedimentation

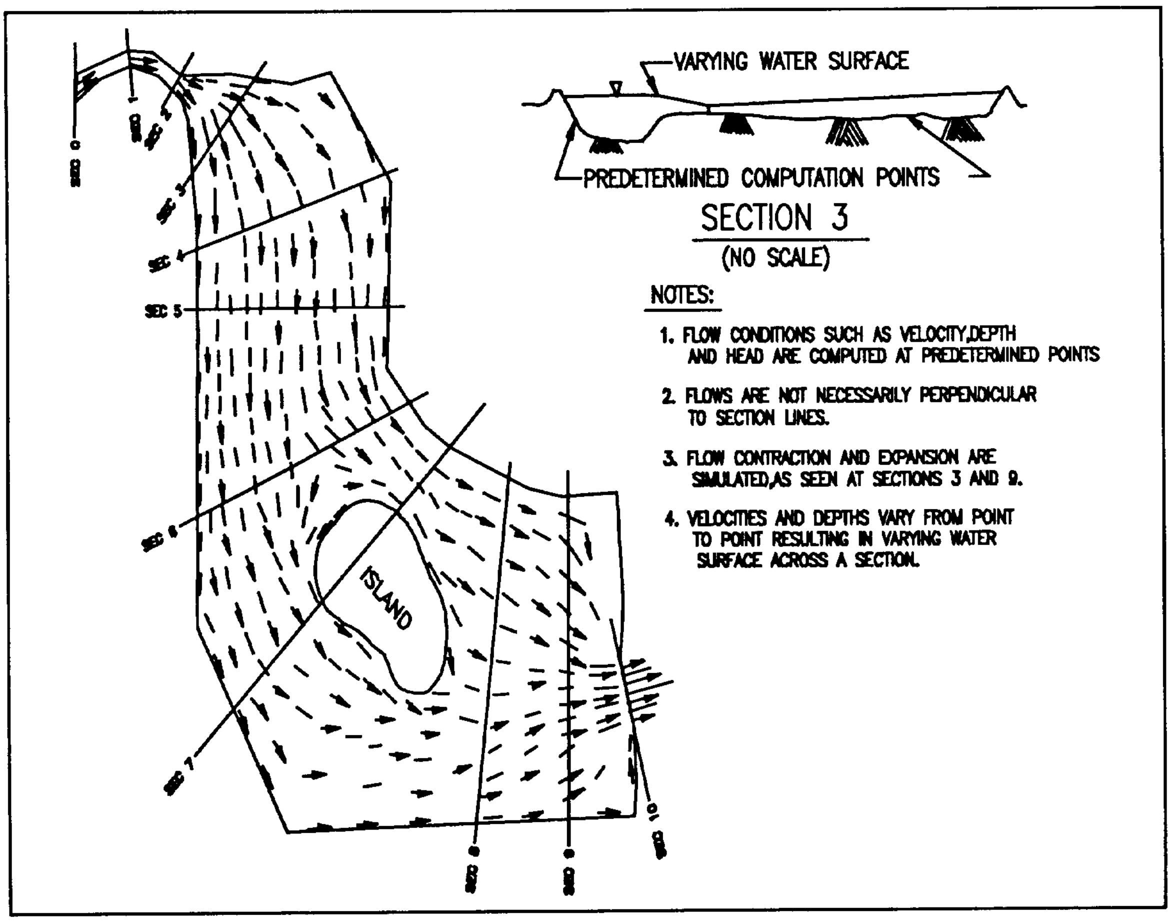


Figure 6-6. 2D flow representation in Cache Creek settling basin

or erosion. If the vertical depth components cannot be averaged and a 3D simulation is necessary, physical models must be employed.

6-6. Mobile Boundary Hydraulics

Mobile boundary analysis is necessary when the assumptions of a rigid boundary are no longer valid. For most streams, a rigid boundary assumption is acceptable for a design flood, as channel and overbank geometry are typically slow to change in response to the sediment transport characteristics causing scour and deposition. Over time, however, the stream does respond to changes in its sediment regime by adjusting its cross-sectional geometry, stream slope, bed material composition, sediment load, etc. Mobile boundary analysis is thus generally concerned with the longer-term trends over the life of a flood reduction or navigation project. The complexity level of a mobile boundary analysis is similar to that of a one-dimensional unsteady flow analysis (USACE 1991a).

a. Basic principles.

(1) Assumptions. The most common mobile boundary analysis incorporates a number of important assumptions, including:

(a) The analysis is one-dimensional (single water surface elevation at each point).

(b) The channel slope is small.

(c) Sediment-water density is constant.

(d) Manning's n value applies.

(e) Gradually varied flow occurs along the stream channel.

(2) Models. These assumptions may be incorporated into mobile boundary models. The models commonly combine a gradually varied steady flow analysis with sediment transport calculations at the end of each flow period. Changes in channel geometry are calculated before starting the next computation period. Computations would normally take place over several years of discharge data to identify long-term trends occurring in channel geometry and water surface elevations.

(3) Results. The model may be calibrated and operated to give information like channel invert and water surface profiles. The most valuable information is the identification of trends and the comparison of the effects of a flood mitigation component on the sediment regime. Model results can be used to evaluate and compare with- and without-project conditions. A typical application would determine how fast a channel improvement, or a reservoir, loses capacity due to sediment deposition.

b. Basic data. A mobile boundary analysis typically requires the most data of any of the methods of analysis described in this chapter, necessitating hydrologic, geometric, and sediment information.

(1) Hydrologic data. Discharge data are needed for all flow periods, from flood to drought. The time duration associated with each of the actual discharges is also necessary. The discharge and time data are often converted from a continuous, smooth hydrograph to a histogram, or bar graph, averaging smaller flows over long time periods. The water temperature is also important, as it has a significant effect on how fast small particles settle in the water column.

(2) Geometric data. Channel cross sections and reach lengths are required, similar to the information necessary for a gradually varied steady flow model. Geometric data are normally less extensive than for a water surface profile analysis, however. Longer distances between sections are tolerable, and bridge sections are not normally included. Manning's n values are used for boundary friction estimates.

(3) Sediment data. The sediment composition of the channel section at each point is needed, with this data coming from borings and/or "grab" samples by the engineer in the field. The amount and composition of sediment flowing in the water column for a wide range of discharges must be determined for the main channel and any significant tributaries. This information is best obtained from actual measurements of sediment load at gage locations, but may be derived in the absence of any real data. The unmeasured, or bed load (that moving within a few inches of the channel surface) must be estimated and included. Geometric and channel sediment composition data require measurement at two or more widely separated time periods to provide calibration information for the sediment transport model.

c. Applications. The primary application of sediment transport models is to evaluate with-project against without-project conditions to determine long-term trends affecting project design and operation and maintenance of the project. Typical applications include:

(1) Determining sediment rate in reservoirs and length of useful life.

(2) Determining rate and location of deposition in channel modifications to estimate frequency of dredging and sediment removal, thereby maintaining design channel capacity.

(3) Determining deposition along a levee over time and the corresponding effects of this deposition on increasing flood heights, thereby decreasing the levee protection.

(4) Maintaining adequate depth at all times at locations where this is important, such as for navigation channels.

(5) Monitoring locations where great changes in channel geometry occur during a flood, such as flow across an alluvial fan.

d. Limitations. Limitations for one-dimensional sediment transport analysis are the same as for one-dimensional unsteady-flow problems. Sediment scour and deposition that cannot be assumed reasonably uniform at a channel section require multi-dimensional or physical model testing. Scour evaluations around cofferdams, navigation locks, or similar structures usually require a higher level of analysis.

6-7. Use of Physical Models

Physical models are employed when mathematical models cannot adequately simulate the full range of effects caused by the component or problem under study. Three-dimensional analysis most often results in physical model testing. These models are normally expensive to build and operate, and require particular engineering expertise to utilize. Typical applications of physical modeling include:

a. Analysis of river navigation improvements on channel geometry and sediment characteristics.

b. Verification/modification of hydraulic design of flood reduction components to minimize operational problems and optimize performance under all adverse conditions.

c. Simulation of navigation through potential hazardous river reaches.

d. Water quality simulations, dispersal of pollutants, and temperature stratification in reservoirs.

6-8. Comparison of Flood Elevation Determination Methods

Although comparisons between the various methods have been made throughout this chapter, additional comparisons are provided in the following tables. Table 6-1 illustrates when the various methods are usually appropriate for different reporting levels, while Table 6-2 gives a rather subjective appraisal of the differences in experience level, time, money, data needs, and computer requirements for the various techniques.

Table 6-1
Model Usage for Hydrologic Engineering Studies

Study Stage	Existing Data & Criteria(1)	GVSF	MB	GVUSF	Multi-D	Physical
Reconnaissance	X	X				
Feasibility		X	X	X(2)	?(3)	
Reevaluation		X	X	X(2)	?(3)	?
DM					X(4)	X(5)

(1) Existing data and criteria = available reports, U.S. Army Engineer Waterways Experiment Station (WES) criteria, regional relationships for depth frequency, normal depth rating relationships, etc.; GVSF = gradually varied steady flow; MB = mobile boundary analysis; GVUSF = gradually varied unsteady flow, multi-dimensional analysis, Physical = physical models (by WES or similar agency).

(2) Use is possible, but unlikely, on most flood control studies.

(3) ? Possible, but very unusual--very dependent on problem being analyzed.

(4) Typically employed to evaluate design performance for a short reach of river, or in the immediate vicinity of a specific project component, or refine the hydraulic design of a project component.

(5) Typically performed to evaluate 3D or other specific conditions where mathematical modeling results are considered inaccurate.

Table 6-2
Qualitative Comparison of Different Analysis Technique Requirements[1]

Analysis Technique	Hydraulic Engineer's Time	Special Technical Expertise Requirements	Computer Requirement	Data Requirement	Study Cost
Existing or simplified criteria	1	None[2]	0,1	1	1
Gradually varied, steady flow	10	None[2]	10	10	50
Gradually varied, unsteady flow	30	Some[3]	20	20	100
Mobile boundary analysis	30	Some[3]	40	30	150
Multi-dimensional analysis	40	Many[4]	100	50	200
Physical modeling	100	Severe[5]	---	100	500

[1] Comparisons among techniques would be as follows: multi-dimensional analysis would require four times the amount of engineer time and five times the amount of data compared to the gradually varied, steady-flow technique.

[2] "Average" hydraulic engineer can adequately handle this technique.

[3] "Average" hydraulic engineer has limited experience in these techniques.

[4] "Average" hydraulic engineer has no experience in this technique, specialized training/assistance by consultants may be necessary.

[5] Would require the use of WES or similar consultant.